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UNUSUAL ENGINEERING FEATURES OF THE AL MALAIKAH TEMPLE, LOS ANGELES

JOHN C. AUSTIN, *Architect*

By R. McC. BEANFIELD, ASSOC. M. AM. SOC. C. E.,
Consulting Engineer

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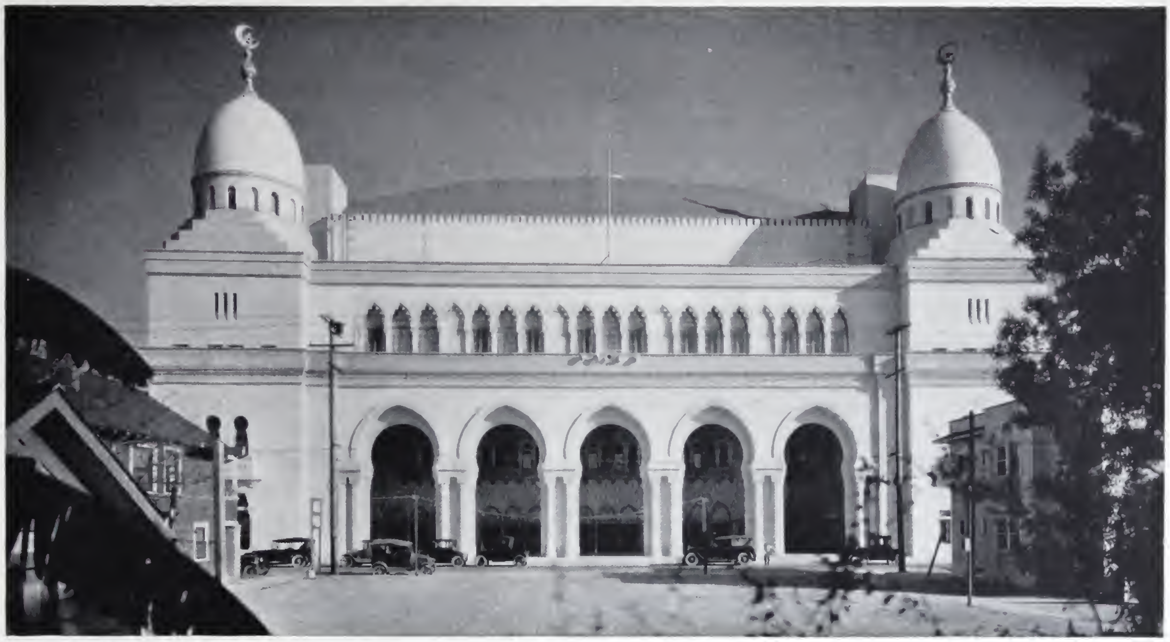

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AL MALAIKAH TEMPLE, LOS ANGELES, CALIF.—JOHN C. AUSTIN, ARCHITECT

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The Proceedings of the American Society of Civil Engineers issue of December, 1927, contained on pages 2645 to 2674 a description of the "Unusual Engineering Features of An Immense Theatre Building" by R. McC. Beanfield, Assoc. M. Am. Soc. C. E. Extracts from this article are published by permission of the author and the American Society of Civil Engineers.—THE EDITORS.

THE Al Malaikah Temple in Los Angeles, locally known as the Shrine Civic Auditorium, occupies nearly three acres and may be divided into three units, as follows: The auditorium, or theatre, with facilities for housing the various Shrine organizations; the banquet hall, with a large basement for public exhibition purposes; and a kitchen wing, with facilities for serving large banquets. The auditorium is 200 ft. wide and 294 ft. deep including the stage, which is 72 ft. deep and 192 ft. wide. The proscenium opening has a clear span of 100 ft. and a height of 37 ft. at the crown. The orchestra pit can accommodate 200 musicians. There are 3,200 seats on the orchestra floor and 3,350 in the balcony. With the exception of 22 seats behind balcony columns, the entire audience has an unobstructed view of the stage. The stage has a seating capacity of 1,700. The auditorium, or theatre, consists of a reinforced concrete frame with exterior concrete curtain-walls. The roof construction consists of reinforced concrete joists, supported on structural steel purlins connected to the roof trusses.

The auditorium balcony risers and treads are of reinforced concrete, supported by eight steel cantilever trusses having a maximum overhang of 45 ft. which, in turn, are supported by a main balcony steel truss. This truss is, doubtless, the largest steel balcony truss yet constructed for the purpose. It weighs nearly 250 tons, and has a clear span of 168 ft. It is of interest that the balcony truss columns were located to displace a single seat on each side of the auditorium nearest the aisles adjacent to the walls, instead of outside the auditorium walls,

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which arrangement would have increased the span 20 ft. The obstructed view for a few seats is relatively unimportant as compared with the increased difficulties in design and greater cost if 20 ft. had been added to the span of the main balcony truss.

As it is necessary to have sufficient clearance through the center part of this main truss for the balcony exit passageways, pin joints and eye-bar diagonals were used instead of the usual built-up members and riveted joints with larger gusset-plates. Riveted joints were used where practicable to obtain general stiffness. Where pins were necessary, nickel steel was chosen, particularly, to reduce the size of the pin-holes in the lower chord.

The balcony truss was delivered to the job in sections, the largest (center lower chord section) weighing 47 tons and being 80 ft. long. The truss was erected on a steel substructure consisting of the same members as were used in the falsework for the roof trusses. The lower chord sections were first set on camber screw-jacks. The web members were next placed, and followed by the top chord sections.

The jack trusses, T-1, tend to equalize the overhang of the cantilever trusses, which project a maximum of 45 ft. beyond the main balcony truss. It was desirable to prevent excessive and unequal deflections of the structural steel trusses supporting the balcony, particularly the cantilevers, and the possibilities of cracking and otherwise shearing the relatively thin monolithically finished 2½-in. slabs and narrow 5-in. risers of the reinforced concrete balcony framing. Hence, it became necessary to design and arrange the balcony trusses so that their deflections would be relatively uniform and slight. Deflections of the various trusses were carefully computed and checked graphically. The maximum computed deflection of the cantilever trusses under dead load was ¾ in. Secondary stresses were carefully computed for the balcony trusses and the roof trusses. Deformations in the main balcony truss, T-2, under full dead load conditions, were partially compensated by the forcing of initial stresses in the truss members during erection. Subsequent strain gauge readings did not indicate a very close relation between the computed stresses and the actual stresses. Those members which showed maximum secondary stresses by extensometer readings were likewise indicative of maximum secondary stresses by the analytical methods. In any event, the strain gauge readings indicated that all stresses were within the maximum safe stress allowable.

It is interesting to note that the strain-gauge

Photo by Mott

THE PROSCENIUM OPENING HAS A CLEAR SPAN OF 100 FT. AND A HEIGHT OF 37 FT. AT THE CROWN. AMPLIFYING SPEAKERS ARE CONCEALED IN THE JEWELED CROWN OF THE ARCH readings on the main steel columns supporting the balcony truss indicated a small moment with a maximum stress of 10,875 lb. per sq. in., which was safe, being about 30% greater than the computed stress. These columns were designed with a very low unit stress, 8,500 lb. per sq. in., to allow for increased stresses due to bending caused by pin friction and other causes, which could not be computed accurately. Each of these columns is braced near its top to the reinforced concrete wall structure by three 10-in. H-struts.

Jack truss T-1 is attached at one end to the main balcony truss by a pin connection in a large steel casting resting on a built-up stool between the web members of truss T-2. The other end is supported on a reinforced concrete wall column. The chords of truss T-1 consist of two 15-in. channels, with 8-in. H web members. The bottom chord in the end panel section of the balcony truss, T-1, was composed of two 8-in. standard eye-bars to avoid clearance difficulties. The pin joint connecting truss T-1 to the main balcony truss, T-2, eliminated any possibility of excessive restraining moments due to the varying deformations of the connected trusses.

There are eight cantilever trusses, four of which are directly supported and connected on the main balcony truss, T-2; the other four are supported and connected at their fulcrum points to the jack trusses, T-1. The center four cantilever trusses are riveted at their fulcrum supports directly to the vertical member of the T-2 main balcony truss, which is common to both trusses.

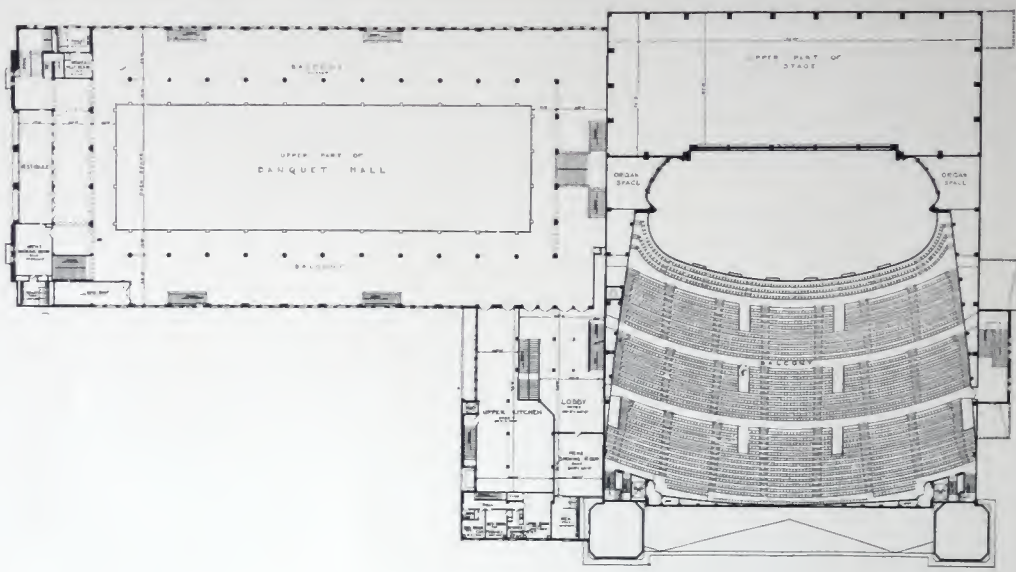
The load on the main balcony truss columns is distributed by a 52 by 52 by 4-in. rolled steel slab to the reinforced column below. It was found to be more economical to use a reinforced concrete pier from the foundations to the first floor level than to extend the steel columns to the footings.

In order to stiffen the ends of the cantilever trusses and reduce their deflections to a minimum, a double web plate girder section was used. During erection, the plate girder sections were given a camber by offsetting the rivet holes in the bottom chord in the joint connecting them to the open web section of the truss. The chords were built of 8 by 3-in. angles, latticed and placed 1 ft. 3 in., back to back, which width added to the lateral stiffness. The anchor, or wall, ends of the cantilever trusses were secured to anchor-bolts on top of reinforced columns.

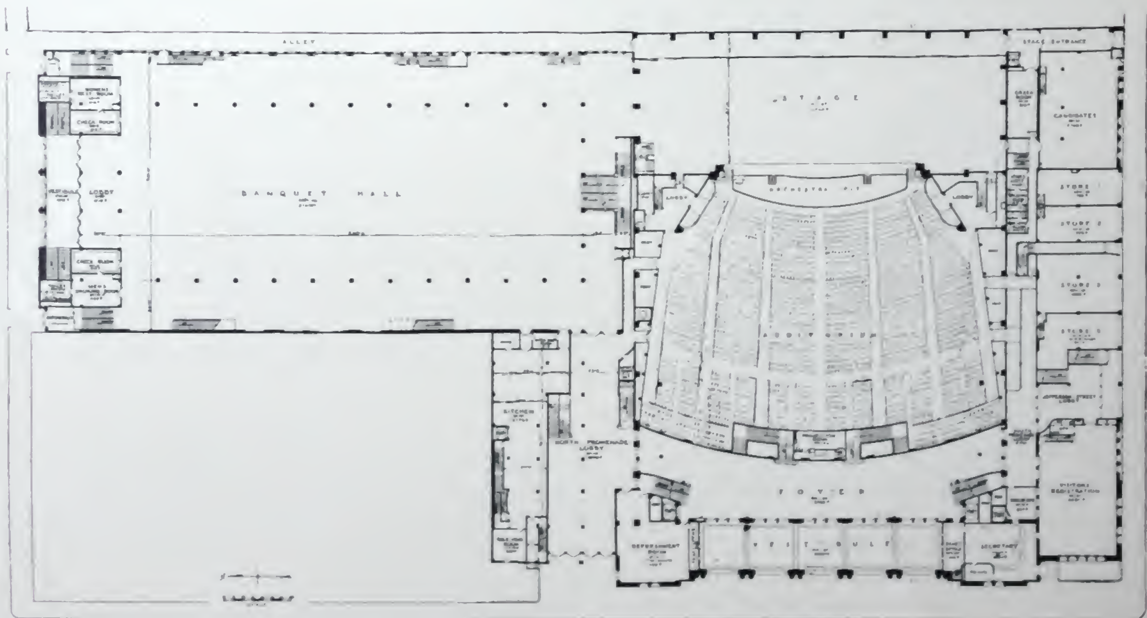
Special attention was given to the design of the general bracing throughout the balcony framing.

Photo by Keystone

THE 22 FT. HIGH CHANDELIER, WEIGHING APPROXIMATELY FIVE TONS, IS CONSTRUCTED WITH A STRUCTURAL STEEL FRAME AND HUNG FROM PIN JOINTS. IT CONTAINS 500 ELECTRIC BULBS, ARRANGED FOR 64 COLOR COMBINATIONS



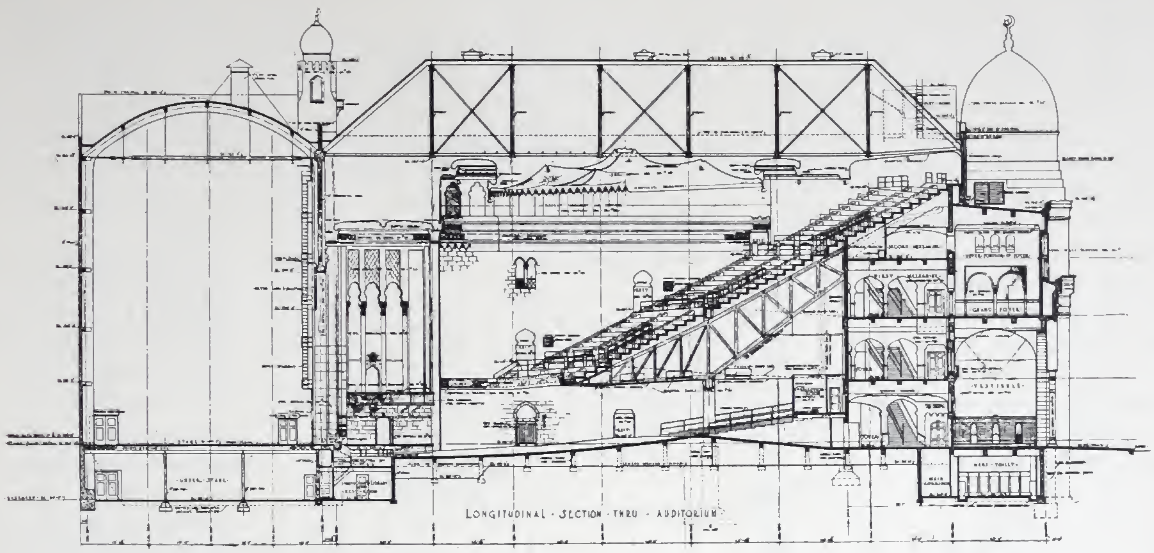
BALCONY FLOOR PLAN



MAIN FLOOR PLAN

AL MALAIKAH TEMPLE, LOS ANGELES, CALIF.

JOHN C. AUSTIN, ARCHITECT



LONGITUDINAL SECTION THROUGH AUDITORIUM
NOTE STAGE ROOF ARCHES WITH GRIDIRON AS TIE BARS

Photo by Mott

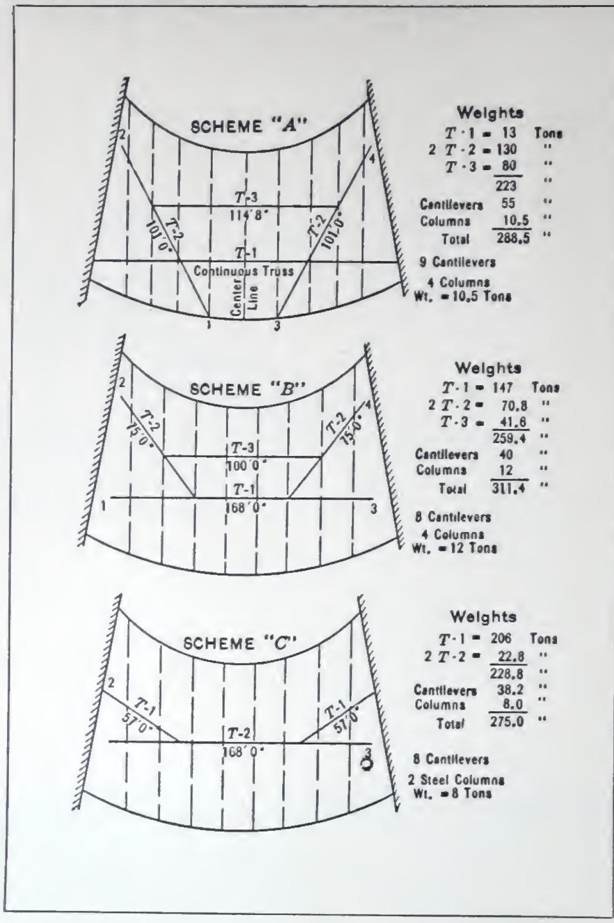
THE ROOF OVER THE BANQUET HALL IS SUPPORTED ON REINFORCED CONCRETE ARCHES OF 90 FT. CLEAR SPAN. BENDING STRESSES IN COLUMNS ARE REDUCED BY THE CANTILEVERED BALCONY. DECORATIONS ARE PAINTED ON CONCRETE

AL MALAIKAH TEMPLE, LOS ANGELES, CALIF.

JOHN C. AUSTIN, ARCHITECT

The cantilever trusses were sway-braced in horizontal and vertical planes by angle cross-bracing and struts in the form of bents. The cantilevers and jack trusses assisted materially in bracing the main balcony truss in addition to the four horizontal boxed latticed struts which braced the upper chord to the rear wall. The four exits passing through the main truss were supported by hanger rods anchored above in the concrete deck of the balcony.

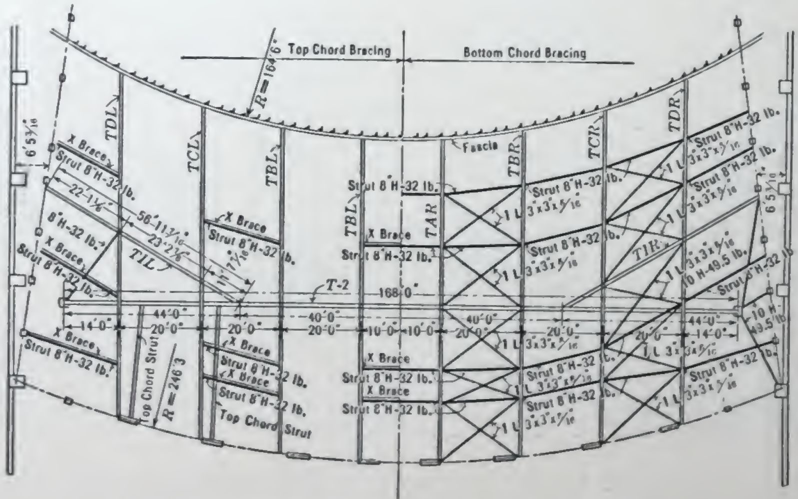
To avoid any possibility of the concrete balcony cracking along the auditorium walls, due to deflections of the supporting trusses, a construction joint was placed in the concrete deck, midway between the auditorium wall and the adjacent cantilever trusses. Although the balcony has been loaded to capacity several times, no cracks in the concrete decking have been observed up to this time.



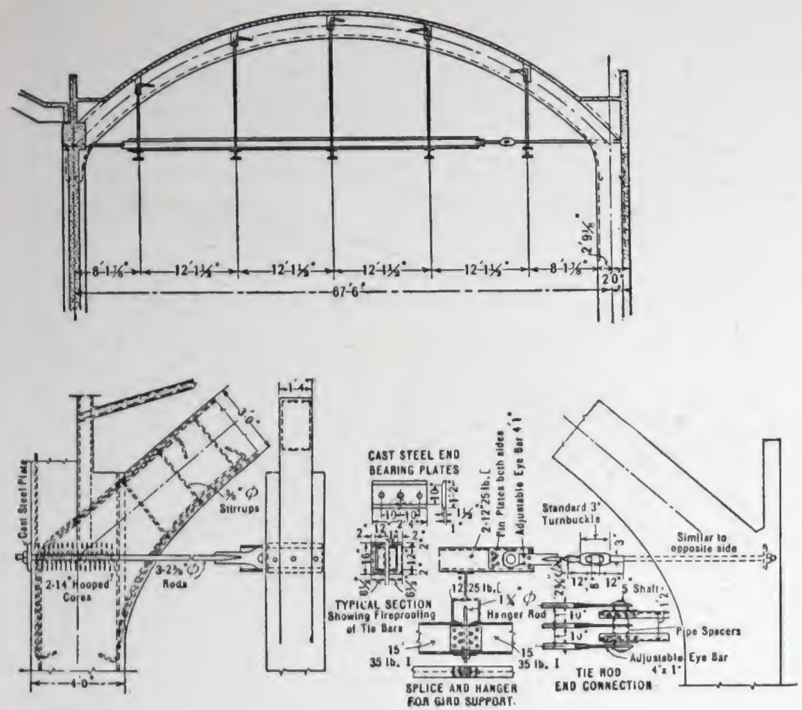
PRELIMINARY DESIGNS AND ESTIMATES WERE MADE FOR THREE DIFFERENT SYSTEMS OF BALCONY TRUSS FRAMING. SCHEME C WAS SELECTED AS BEING THE MOST PRACTICAL, ECONOMICAL AND RATIONAL

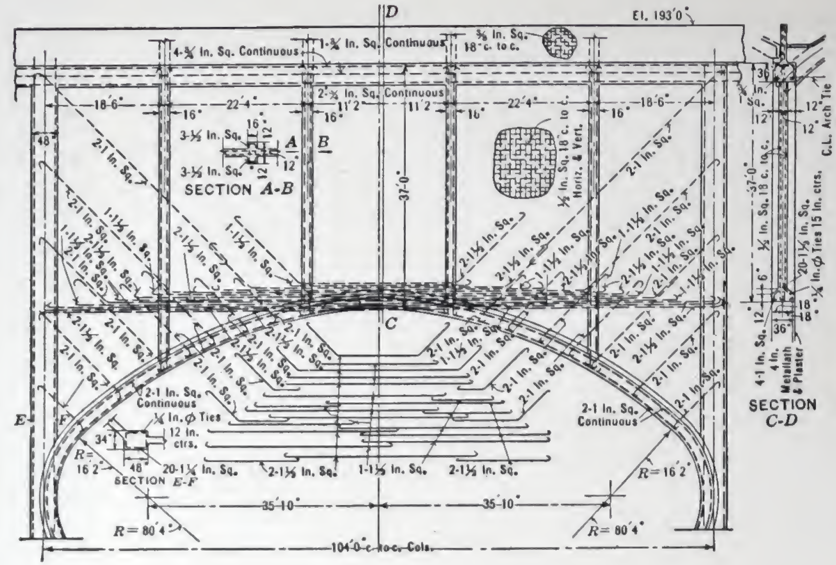
The local building ordinance required a full live load test of the balcony which was accomplished by using metal barrels filled with water. After the test the water was siphoned from the barrels.

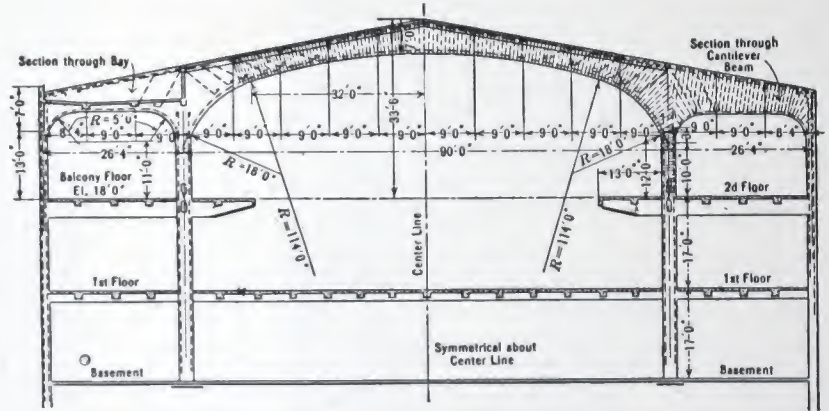
The stage roof arches, eight in number, are spaced 22 ft. on centers. One end of each of the center arches is supported by the proscenium arch wall. The arches were designed as partly fixed. Turnbuckles were placed in the tie bars near the west wall to develop equal stresses in the two channels that form the horizontal tie and to produce a small amount of initial stress in the tie prior to stripping the forms. The gridiron floor, supported on the arch tie-bar channels consisted of flats 4-in. by ⅜-in., spaced 7 in. on centers, a type of construction that resulted in considerable saving as



PLAN SHOWING BRACING OF BALCONY TRUSSES







ABOVE AT LEFT: DETAILS OF STAGE ROOF ARCHES
ABOVE AT RIGHT: DETAILS OF PROSCENIUM ARCH REINFORCEMENT
AT RIGHT: CROSS SECTION OF BANQUET HALL

AL MALAIKAH TEMPLE, LOS ANGELES, CALIF.

JOHN C. AUSTIN, ARCHITECT

MAIN BALCONY TRUSS SHOWING PIN COLUMN CONNECTION AND PIN CONNECTION OF JACK TRUSS TO STEEL CASTING ON MAIN TRUSS. ABOUT 30% OF HOLES IN CONNECTIONS WERE BOLTED TIGHT BEFORE ONE INCH TAPERED RIVETS WERE DRIVEN UNDER 120-LB. PRESSURE

compared with the usual channel sections. The gridiron floor was designed for a live load of 100 lbs. per sq. ft., which has proved ample for stage operating conditions.

The stage roof arches were poured in one continuous operation, commencing from each springing line and working simultaneously and symmetrically toward the crown. The arches and the beam system were poured up to the under side of the roof slab, tee action being obtained by stirrups and bond. By casting the arched roof slab with concrete having a slump of 2½ in., cover boards were found to be unnecessary. The roof arches were cast with a concrete mixed in the proportion of 1 part cement to 4.5 parts aggregate.

The proscenium arch, constructed of reinforced concrete, has a clear span of 100 ft. This arch was computed as a girder with partly restrained ends with diagonal tension stresses resisted by diagonal bars. In order to increase the lateral stiffness, pilasters were placed in the web under each roof arch similar to the stiffeners in plate girder construction. A flanged head, or tee, in the top of the proscenium girder also adds to its lateral stiffness.

Due to the great unsupported height of the reinforced concrete walls and columns surrounding the

Photo by Dwyer

OUTER ENDS OF BALCONY CANTILEVER TRUSSES WERE SUPPORTED BY CABLES FROM ROOF TRUSSES BEFORE RIVETING. PLATE GIRDER ENDS OF CANTILEVERS ARE USED TO REDUCE DEFLECTION AT OUTER END stage, horizontal diagonal brace beams were installed at vertical intervals in all four corners of the stage enclosure. The stage walls were designed as two-way slabs to resist a wind pressure of 30 lb. per sq. ft. The stage columns were computed for direct stress and bending due to wind, and for partly fixed conditions of the roof arches.

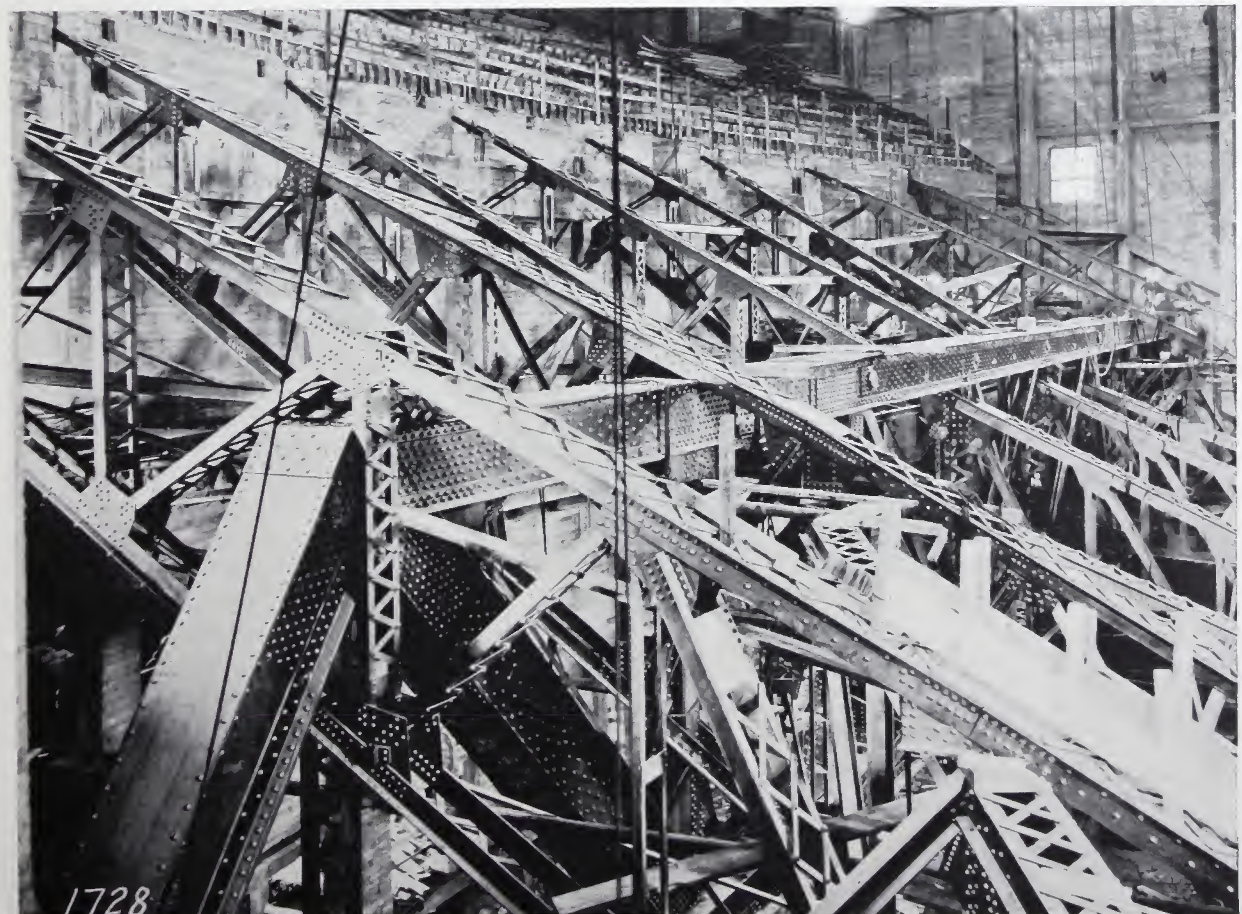
The stage floor was designed for a live load of 250 lb. per sq. ft. The framing consists of standard steel I-beams supporting a wood-joist system. All steel connections were bolted so that any particular section of the floor could be removed without affecting the adjacent parts.

The structural steel roof trusses over the auditorium, each weighing 60 tons, have a clear span of 192 ft. and rest on reinforced concrete columns.

The auditorium proper was constructed with double walls on each side to resist earthquake forces and provide duct space for the ventilating systems. The inner walls of the auditorium are metal lath and plaster, supported by reinforced concrete columns and beams. This skeleton frame forms a system of braced bents with the outer walls and columns, which support the roof trusses.

Due to the large area of the auditorium and the long-span roof trusses, it was necessary to divide

MAIN BALCONY TRUSS SHOWING INTRICACY OF CONNECTION DETAILS AND PIN CONNECTION OF JACK TRUSS TO STEEL CASTING ON MAIN TRUSS. ELECTRIC WELDING WAS USED TO OBTAIN METAL-TO-METAL CONTACT WHERE INCLINED SURFACES MADE IT DIFFICULT TO MILL ACCURATELY



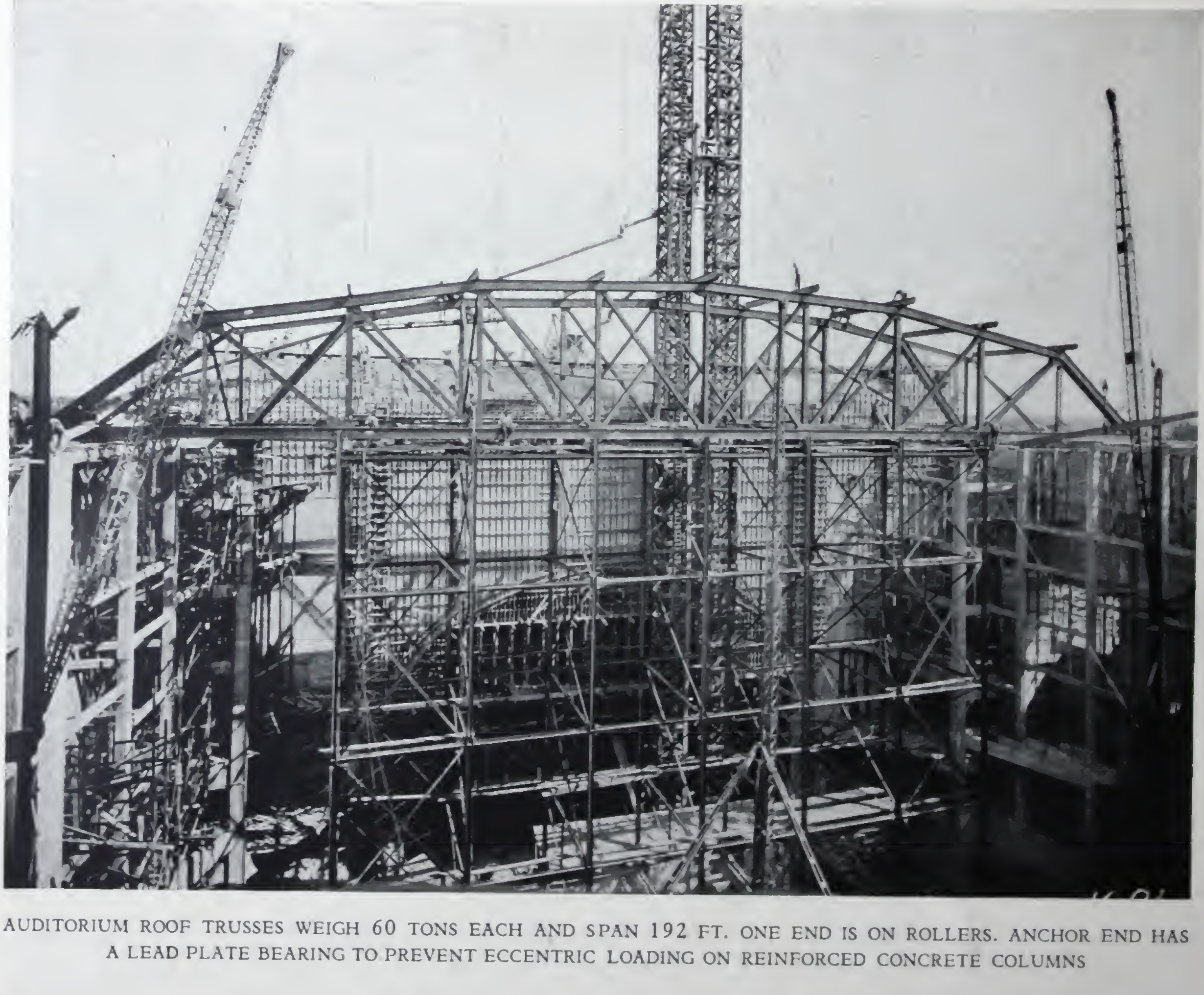
MAIN BALCONY TRUSS AND CANTILEVERS. CANTILEVERS WERE HOISTED TO CAMBERED POSITION. HOLES WERE DRILLED AND MATCHED BEFORE RIVETING. NOTE PIN JOINTS IN UPPER CHORD the roof area into sections by expansion joints to prevent excessive deformations due to heat changes.

The south ends of the roof trusses were anchored; the north ends were set on nests of rollers. This provided an excellent structural connection for earthquake resistance. The roller bearings allow the roof and supporting trusses to move (horizontal translation) independently of the north supports with temperature variation, which is also a favorable structural condition of flexibility for relieving seismic stresses. If the roof trusses had been anchored at both ends, they would act like battering rams, tending to pull or push over the supports, particularly if the periods of vibration were different for the opposing supports.

To prevent excessive edge pressure on the reinforced concrete columns at the anchor end of the trusses, due to the slope at the end panel joint, a $\frac{1}{8}$-in. lead plate was inserted between the bearing and shoe-plates. Lateral and vertical deformations, based on Williot diagrams, checked very closely with the actual horizontal movement over the rollers (maximum, $\frac{3}{8}$-in.) and the vertical deflections (maximum, $2\ ^{3}/_{16}$-in.).

The continuous reinforced concrete girders, supporting the banquet hall roof, were designed as a part of a fixed frame. The soffits of the roof girders were curved to represent Saracenic arches, which added to the depth of the girders over the interior columns where large negative moments existed. The use of balcony cantilever girders tended to reduce the moments in the columns. The rear, or wall, columns were required to resist an uplift. The anchor arms of the continuous girders were projected through and above the roof, being hidden from the interior.

John C. Austin, F.A.I.A., was the architect. The writer, as chief engineer for the architect, was responsible for the structural design and the design of the heating and ventilating systems.



AUDITORIUM ROOF TRUSSES WEIGH 60 TONS EACH AND SPAN 192 FT. ONE END IS ON ROLLERS. ANCHOR END HAS A LEAD PLATE BEARING TO PREVENT ECCENTRIC LOADING ON REINFORCED CONCRETE COLUMNS